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ALTERNATIVE SEWER STUDIES

by

Urban Systems Research & Engineering, Inc.
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16. ABSTRACT

An evaluation of design and operational aspects for two popular alternative sewering techniques, i.e., pressure and small-diameter gravity sewers, was performed. The results provided insights useful to designers of these systems with regard to sulfide concentrations in these sewers and hydraulic designs for downhill runs.

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FOREWORD

The U.S. Environmental Protection Agency is charged by Congress with protecting the Nation's land, air, and water systems. Under a mandate of national environmental laws, the agency strives to formulate and implement actions leading to a compatible balance between human activities and the ability of natural systems to support and nurture life. The Clean Water Act, the Safe Drinking Water Act, and the Toxics Substances Control Act are three of the major congressional laws that provide the framework for restoring and maintaining the integrity of our Nation's water, for preserving and enhancing the water we drink, and for protecting the environment from toxic substances. These laws direct the EPA to perform research to define our environmental problems, measure the impacts, and search for solutions.

The Water Engineering Research Laboratory is that component of EPA's Research and Development program concerned with preventing, treating, and managing municipal and industrial wastewater discharges; establishing practices to control and remove contaminants from drinking water and to prevent its deterioration during storage and distribution; and assessing the nature and controllability of releases of toxic substances to the air, water, and land from manufacturing processes and subsequent product uses. This publication is one of the products of that research and provides a vital communication link between the researcher and the user community.

This report provides additional design and operational information on two of the most effective and widely applied alternative sewer systems, i.e., small-diameter gravity and pressure sewers, over and above previously published reports. The information provided herein will assist system designers and operators to avoid and/or rectify problems resulting from sulfides and downhill hydraulics which could otherwise represent major impairments to the successful application of these technologies.

Francis T. Mayo, Director
Water Engineering Research Laboratory

ABSTRACT

This study deals with design and operational considerations not practically addressed in previous reports on small-diameter gravity (SDG) and pressure sewer systems. These issues are sulfide generation and hydraulic design of downhill runs.

Sulfide generation is demonstrated to be a problem in both types of pressure sewers, i.e., grinder-pump (GP) and septic tank effluent pump (STEP), and SDG sewers. The highest values were found in pressure sewers, but sulfides were also found in significant concentrations in the SDG sewer. Other than the verification of substantial sulfide concentrations in these types of sewers, no quantitative modeling of sulfide profiles in sewers was attempted.

Early pressure sewer designs were based on water supply system design technology. Those systems which were not generally flat or rising in profile have experienced numerous problems due to air introduction and accumulation in the mains. This study details the development and full-scale results of an improved design approach for downhill portions of pressure and SDG sewers to minimize air accumulation and its attendant operational problems.

This report was submitted in partial fulfillment of Contract No. 68-03-3057 by Urban Systems Research and Engineering, Inc. under the sponsorship of the U.S. Environmental Protection Agency. This report covers a period from March, 1981 to July, 1983, and work was completed as of September, 1983.

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SECTION 1

INTRODUCTION

Small communities in need of new or expanded sanitary sewers are faced with a severe financial burden. Low densities and unfavorable geological situations increase per capita costs of conventional sewers, which often account for up to 80% of the total capital costs of a new wastewater management system (9).

Conventional sewers are expensive. In order to insure that raw sewage flows freely conventional sewer systems use large diameter pipes set in the ground at minimum slopes. Often pumping stations are required as well. Extensive excavation is usually necessary to achieve the desired slopes. Flat terrain, high groundwater, and waterfront areas all add to construction costs and difficulties. Finally infiltration and inflow (I&I) cannot be eliminated in large pipes. The added wastewater volume and solids mean that the treatment plant must have a greater capacity than would be required to only treat the sewage.

Alternative approaches to sewering that address some of the problems encountered with conventional systems can reduce collection and treatment costs. Three types of alternative sewers are discussed below:

- o small diameter gravity sewers (SDGS),
- o pressure sewers and
- o vacuum sewers.

Small diameter pipes reduce excavation and construction costs. Vacuum or pressure sewers offer the dual advantages of small diameter pipes and the ability to follow the natural topography without risk of clogging which further lowers excavation costs. Furthermore all three of these sewers provide a system virtually sealed against infiltration inflow.

The two major types of pressure sewers are grinder pumps (GP) and septic tank effluent pumps (STEP). These two systems differ in the onsite equipment, layout, and the quality of the wastewater conveyed to the pressure sewer. In GPs solids are ground to a slurry and discharged through pressure lines. In STEPs wastewater from a home first flows into a septic tank from which treated effluent is pumped to pressurized lines.

Vacuum sewers use a central vacuum source to constantly maintain a vacuum on small-diameter collection mains. Periodically the pressure differential created by the vacuum source draws a slug of sewage from a holding tank at each home into the line. When sufficient volume of sewage is collected at a central vacuum station, it is pumped to the treatment plant or main interceptor.

Like the STEP system, a small-diameter gravity (SDG) sewer is used with individual septic tanks. Because solids are removed by the septic tank, pipes as small as 4 inches in diameter can be used at very shallow slopes without risk of clogging. The effluent requires little or no pumping and flows by gravity to the treatment facility.

Despite their many advantages, several concerns have been raised about alternative sewers. The most important of these potential problems are:

- excess sulfides generation;
- two-phase flow in pressure and SDG sewers.

Sulfides generation effects all types of sewers. The problem manifests itself in unpleasant odors and corrosion produced by hydrogen sulfide. Years of experience have gone into the design of conventional sewer systems to minimize sulfide generation. In 1974 EPA published a design manual setting forth the best engineering practice (7).

Based on experiences with sulfides in conventional sewer systems, two main concerns have been raised with regard to alternative systems. Is the septic effluent in SDG's and STEP systems more prone to sulfides generation, and does the anaerobic nature of pressure and SDG sewers contribute to sulfide generation?

Two-phase flow refers to a hydraulic problem of particular concern in pressure systems. In downhill sloping sections of pressure sewers, gas bubbles present in the pipeline can adversely affect flow. The typical solution is to install air release valves at summits within the pipeline. However, in many cases, this action does not work and additional steps must be taken to solve the problem.

SECTION 2

CONCLUSIONS

Although a significant number of pressure and small-diameter gravity (SDG) sewers have been designed and constructed there remain some significant gaps in understanding the technology. These studies provided some insights as to two of the major technology gaps. The major conclusions are:

1. Grinder-pump (GP) systems can produce sulfides at a rate of 3 to 4 times that of septic tank effluent pumping (STEP) systems and about twice that of conventional sewer force mains due to the high organic strength of the wastewater.

2. There appear to be unexplained losses of sulfide and gains in dissolved oxygen in STEP systems based on analyses performed in this study and previous data for septic tank effluents.

3. Both pressure systems, i.e., GP and STEP, can be expected to have some sulfide concentration in their wastewaters, with value varying from 1 to 14 mg/L, based on this study.

4. GP sulfide concentrations will generally increase in the direction of mainline flow, but random locations of service lines and branches may mask this trend.

5. Concentrations of sulfides in pressure sewers cannot as yet be quantifiably predicted, due to the empirical nature of the available equations and their derviation for weaker conventional wastewaters.

6. SDG sewers should not be designed to minimize pipe sizes, to flow full for substantial periods, or to proliferate substantial periods, or to proliferate substantial inundated sections of mainline if sulfide minimization is desired.

7. For conventional gravity sewers, equilibrium sulfide concentrations result from long pipe segments with relatively uniform conditions. Applying the equilibrium equation for conventional gravity sewers to SDG sewers results in concentrations comparable to those observed in SDG sewers. These concentrations are much lower than the levels reported to occur in septic tanks.

8. Due to the phenomenon of #7 above, SDG sewers appear capable of producing terminal wastewater sulfide concentrations lower than

those of pressure sewers.

9. Conventional placement of air-release valves at high points of a pressure sewer system does not preclude the entrainment of air and results in headlosses greatly exceeding design calculations.

10. In downhill runs where the pressure main intersects the dynamic hydraulic grade line (HGL) a hydraulic jump is formed which generates gas bubbles to downstream segments of the main.

11. Placement of sewage-type automatic air release valves at points at least 14 pipe diameters below hydraulic jump locations was effective in removing entrapped air and reducing headlosses to near theoretical levels.

12. Backpressure sustaining valves were found to be inadequate for control of downhill hydraulics in the pressure sewer owing to high capital cost, intensive maintenance requirements and unreliable operation.

13. On downhill runs with irregular terrain which provide numerous opportunities for the formation of smaller hydraulic jumps standpipes were found to be inexpensive and reliable. The standpipes employed large diameter downlegs to prevent the escape and conveyance of air bubbles into the downstream segment of the mains and automatic air-release valves at their summits to expel the trapped gases.

14. Soil absorption beds were successfully used for the vented gases from the air-release valves to prevent odors from hydrogen sulfide.

SECTION 3

RECOMMENDATIONS

There is a need to improve the designer's capability to predict sulfide concentrations in pressure and SDG sewers. To accomplish this end comprehensive studies of sulfides should be made from the septic tank to the terminus of a number of these systems to identify the gains and losses of sulfide concentration, to quantify the mechanisms responsible, and to develop predictive equations. Once this is accomplished, a study to develop a corrosion-based methodology for evaluating the alternatives of designing a transition station or modifying a receiving conventional sewer should be undertaken. Such a methodology would provide a quantitative solution to one of the major design obstacles in the design of pressure and SDG sewers which terminate at larger conventional sewers.

There is also a need to more quantitatively assess the need for location of, and design and O/M requirements for air-release valves in pressure and SDG sewer systems, along with the use of soil absorption and other low-maintenance odor control methods appropriate for these alternative sewers.

SECTION 4

SULFIDE GENERATION IN ALTERNATIVE SEWERS

INTRODUCTION

Hydrogen sulfide (H_2S) is a gas that can be found in any type of sewer. Hydrogen sulfide and other sulfur-containing substances such as thiosulfates can cause two main problems in sewers. They can be a source of noxious odor, and they can cause corrosion. H_2S is also very toxic. Workers have been known to succumb to H_2S poisoning when concentrations were as low as 300 ppm in the sewer air.

H_2S is formed by the action of anaerobic bacteria on sulfates dissolved in the wastewater. These dissolved sulfates may be indigenous to the water supply, or be themselves the product of microbial breakdown of waste proteins. Since most sewers contain some oxygen, sulfide generation can only occur in a relatively small part of the sewer that is anaerobic: the layer of slime growing on the walls. Sulfides are formed in a narrow band within the slime -- deep enough so that oxygen does not penetrate, yet shallow enough so that the bacteria can still obtain the organic nutrients they need to survive.

The amount of sulfides generated depends on the amount of sulfate in the water, pH, the available organic carbon, and the thickness of the slime layer. The thickness of the slime layer depends in turn on the composition and abrasiveness of the sewage, and the velocity of the flow.

H_2S concentrations will only increase if the gas is created faster than it is destroyed. Under most circumstances, the sulfides formed within the slime are reconverted back to sulfates by aerobic bacteria living on the slime surface. Because of this, H_2S levels do not usually build up in sewers. Gas concentrations will only rise when the DO levels in the sewer are less than .1 mg/L.

Designers of sewer systems have had to cope with sulfides generation in conventional designs for many years. Some researchers have theorized that small diameter pipes, pressurization, septic tank pre-treatment, grinder pumps and frequent intersection with large diameter gravity collectors amplify the problems of sulfide generation. This section presents the reasons why differences in sulfide generation do or do not exist, and identifies gaps in our current knowledge.

Designers have developed many ways to control sulfides. The techniques that have proven effective involve either preventing the anaerobic

conditions conducive to sulfide formation or limiting exposure of the waste stream to the air where sulfides can be released as a gas.

This portion of the study involved onsite sulfide monitoring in three different alternative sewer systems, review of the monitoring results by Richard D. Pomeroy, an internationally known expert in the field, and finally, a simple analysis of how the physical factors that affect sulfide generation in conventional sewers compare to alternative sewers.

SULFIDES GENERATION IN THREE DIFFERENT ALTERNATIVE SEWER SYSTEMS

It is difficult to generalize across the range of alternative systems. This report does not attempt a comprehensive discussion of all types, but rather reviews sulfide conditions found during this study in three separate systems.

Quaker Lake Pa. Grinder Pump System

The Quaker Lake sewer system consists of two separate lines serving 39 and 71 residences, respectively, carrying a total of 18,000 gallons/day. Each house has a wet well in which a grinder pump is located. As sewage fills the wet well, it activates the grinder pump which forces the waste into the pressure mains. The lines are designed to flow under intermittent pressure. The two lines flow to two 750,000 gallon aerated lagoons. The system was installed in 1977 and there have been no reported corrosion or odor problems since that date.

Over a two day period in August 1981, four rounds of samples were taken at six stations along the north branch of the system which serves 39 homes. The samples were analyzed for total sulfides, dissolved sulfides, sulfates, BOD, temperature, and pH. The dissolved sulfide tests were made on samples preserved by the use of zinc salt, which converts dissolved sulfide into insoluble zinc sulfide. This error does not impair the value of this report for its intended purpose because insoluble sulfide is usually only a few tenths of a mg/L in domestic sewage.

The average sulfide concentrations (mg/L) at six stations along the collector, starting at the upstream station were: 9.4, 10.1, 6.7, 8.9, 5.4, 9.0. (Sulfide concentrations in sewers are typically erratic). The sewage temperatures averaged 18.4°C. The pH averaged 7.06.

For sulfates it was found, in line with experience elsewhere, that the results obtained by the standard method normally used for water analyses gave unreliable results when applied to wastewaters. The reasons are not clear, but until dependable methods are found for determining sulfate in sewage, the test should be omitted. The sulfate concentration has little to do with the rate of sulfide generation, but sometimes it is a limiting factor on the amount of sulfide that can be produced.

BOD determinations showed erratic results. The average was 390 mg/L, which is not exceptional for ground domestic sewage near the source.

Glide, Oregon: Septic Tank Effluent Pump System

The sewage first passes through septic tanks and then is pumped into the pressure main and thence to the large oxidation ponds of the City of Glide. There are numerous pressure relief valves. The profile is generally downhill, but there are grade reversals. Falls occur within the mainline at five locations, where oxygen may be absorbed. Despite the fact that the system is not airtight, there have been no odor or corrosion problems since this system was installed in 1975.

Tests for sulfide, BOD, temperature and pH were made at four sampling stations along the main trunk. On September 23, two rounds of tests were started at 4:45 and 7:20 a.m. and on September 24, rounds were started at 6:05 and 7:10 a.m., each round requiring about two hours.

There was not a significant difference between the earlier and later tests. The average measured sulfide concentrations for the four stations, starting at the uppermost, were 7.1, 7.2, 9.2, and 8.5 mg/L. Average temperature of the sewage was 18°C. The pH averaged 6.95. BOD values appeared to reduce with collection system length, while both total and dissolved sulfides increased.

Westboro, Wisconsin: SDG System

This is a complex system, including small diameter gravity (SDG) sewers, a pump station and pressure main, and septic tanks pumping into small diameter pressure mains. Single sulfide tests are reported for samples taken in August from nine manholes. Sulfide concentrations ranged from 1.25 to 3.9 mg/L averaging 2.7 mg/L.

It would be difficult to draw any generalization from a small number of tests, or even from a considerably greater number, in such a complex system.

FURTHER DATA COLLECTION

Sewage is a highly variable material, both in time and place. It is difficult to quantify all of the factors that influence the rate at which a wastewater stream will generate sulfide. The "practical" approach to finding out how various system designs influence sulfide conditions by the procedure of measuring sulfide concentrations in existing systems is not really a very practical approach unless a very large number of systems are studied, with observations being made on many variables other than those reported.

Two essentially different sources of sulfide may exist in these systems: septic tanks and sewers. There can be wide differences in the

condition of septic tank effluents. In one pressure system (in California) the effluents of septic tanks carry an average of about 50 mg/L of sulfide. By contrast there is a large Third World city in which all of the buildings have septic tanks. Only a small part of the city is sewered. Elsewhere in the city, the effluents discharge to channels and storm drains. No more than a few tenths of a mg/L of sulfide was found in any of those flows. This was very puzzling until it was realized that the regular pumping of the tanks was only theoretical; in reality they are very rarely pumped. When a septic tank is full of sludge, the water maintains a channel throught the tank. Because of the slowness of molecular diffusion, very little sulfide can diffuse out into the wastewater stream.

In an alternative system that employs septic tanks, a comprehensive view of sulfide conditions would include information on the sulfide concentrations in the septic tank effluents. Also, transit times of the sewage in the pipelines should be observed by use of dyes. Sulfide build-up rates could be compared using equations such as those presented in the EPA Sulfide Control Manual.

Altogether, it appears that it would be a formidable task to extend this project to a degree that would support any quantitative generalizations. It is unlikely, as these three cases point out, that any lesser effort would produce statistically significant data.

THEORETICAL COMPARISON OF SULFIDE GENERATION IN ALTERNATIVE AND CONVENTIONAL SEWERS

Nearly all of the sulfide produced in small pipes comes from the slime layer on the wall of the pipe. The amount of sulfide produced upstream from a given point is proportional to the upstream area of pipe wall. If the houses contributing flow to the pipeline are more or less uniformly distributed, the flow in the pipeline is expected to be roughly proportional to the distance. If the pipe were uniform in diameter, the wall area, and hence the amount of sulfide, would increase in porportion to the distance, and the sulfide concentration would remain about the same. Actually, the diameter of the pipeline in the Quaker Lake system is a little greater at the downstream end, so the sulfide concentrations might increase slightly toward the end. The randomness of the flow prevents an accurate comparison, but it can be said that there is no clearly demonstrated trend.

From various research studies on this process in conventional sewer pressure mains, an empirical equation was derived to show the expected rate of sulfide build-up in the slime layer on the pipe walls and in the waste stream:

$$\frac{d[S]}{dt} = \frac{3.28\ M\ [EBOD]\ (1 + 0.48\ r)}{r}$$

in which:

$\frac{d[S]}{dt}$ = rate of change of total sulfide concentration, mg/L - hr

M = specific sulfide flux coefficient, m/hr

EBOD = effective BOD_5 = BOD_5 $(1.07)^{(T-20)}$ (mg/L)

T = temperature, °C

r = hydraulic radius = d/4 (m)

3.28 = conversion factor (ft/m)

(1+0.48r) = empirical factor for sulfide production in the stream

Where the dissolved oxygen concentration is not more than a few tenths of a mg/L, M is normally about 0.001. Sulfide buildup rates cannot be predicted precisely, because the quantities measured by the BOD test are not precisely proportional to the sulfide generating nutrients. The calculation, using the coefficients suggested, yields results that will exceed the actual buildup most of the time. The currently available data are not adequate to support a calculation of the expected sulfide concentrations in the alternative systems that were studied.

The factors which contribute to (and can limit) sulfides production include:

- o availability of nutrients (EBOD),
- o sulfates, presumed to be available in sufficient concentration,
- o favorable temperature conditions,
- o no inhibiting biological conditions, and
- o anaerobic conditions.

Assuming the last three conditions are met, the maximum amount of sulfide which can be generated, or the Sulfide Potential, is dependent on the quality of the influent wastewater, the hydraulic conditions of the sewer and temperature. Special attention should be given to sulfide generation in areas with high sulfate concentrations in the water supply.

The rate at which sulfate is converted to sulfide is dependent on

the conditions within the sewers. The rate of change of sulfide concentration in pressure sewers is expressed by the above empirical formula. It is important to note that this formula is based on observation of sulfide generation in conventional system force mains. Since the formula is empirical, it is only applicable for conventional force mains. However it is interesting to review this formula and determine if what is known about the parameters affecting sulfides generation in conventional force mains enables us to forecast significantly different patterns in pressure sewers.

In comparing the basic pressure sewer concept with the equation and other information about the technology, the following appear to be reasonable assertions:

1. Based on EBOD, the rate of sulfide production in GP systems is about 3 to 4 times that in STEP systems and nearly twice that in conventional force mains, all other conditions being equal.

2. If septic tank effluent sulfide concentrations are as high as reported elsewhere (12), i.e., 20-100 mg/L, the fact that in-system and system effluent concentrations are reported herein and elsewhere (1) (2) to be in all cases less than 15 mg/L and normally less than 10 mg/L, there is a consistent loss of sulfide in STEP systems which is not yet obvious. These systems also consistently display a low-level dissolved oxygen (<0.5 mg/L), the source of which is not clear. More thorough sampling and study are required to explain these phenomena.

3. Grinder-pump systems generally receive a wastewater which is devoid of significant sulfide concentration, but consistently displayed in the systems evaluated herein a minimum mainline sulfide concentration of 1.0 mg/L, progressing higher in the direction of mainline flow to values as high as 14 mg/L. These results are consistent with the predictive equation, although not predictable due to varying flows and input locations.

The basic equation for the change of sulfide concentration with time for gravity sewers employs the same parameters as the above equation, without the empirical factor for sulfide production in the stream, in the first term and a sulfide reduction or loss term. When equilibrium conditions occur in the gravity sewer, e.g., a long segment with continuity of conditions affecting sulfide concentration, the gain and loss terms become equal and the equation is expressed as follows (13):

$$S_e = \frac{C_1[EBOD]\,P}{(SV)^{0.375}\,b}$$

in which

S_e = equilibrium sulfide concentration, mg/L

C_1 = coefficient, in English units = 0.00052

S = slope

V = velocity, ft/sec

P = wetted perimeter, ft

b = liquid surface width, ft

Under a given set of hydraulic conditions, sulfides will approach (Se) as an asymptote, either in an increasing or a decreasing mode (13).

Based on the equation for conventional gravity sewers and other known information on SDG sewers, the following statements appear reasonable:

1. SDG sewers should not be designed to minimize pipe sizes or to flow full for substantial periods.

2. The mainline sulfide concentrations found at Westboro which are substantially lower than reported septic tank effluent levels (12), are consistent with the above equation and sewer reaeration. Whether these concentrations are representative of other SDG sewers due to its unique design with multiple manholes remains to be determined, but the potential for lower sulfide concentrations in SDG sewers in comparison to pressure sewers exists.

The parameters of the equation are EBOD (which is directly proportional to BOD_5 and Temperature), M, and empirical constant, and r, the hydraulic radius.

BOD_5	Grinder-pump pressure systems have BOD_5 concentrations of about twice that in conventional systems. STEP systems will have about half the BOD of conventional systems.
Temperature	Should be virtually the same for all sewer types, although STEP wastewaters might be slightly cooler due to lengthy residence time in interceptor or septic tanks.
Hydraulic Radius	Smaller pipes present a higher slime area to wastewater ratio. This increases the rate of sulfide production.

It then follows that designers of alternative systems should to the extent possible follow good design practice to minimize sulfides generation where possible and, where not possible, take appropriate steps to minimize the adverse impacts of odors and corrosion.

It is interesting to review these recommendations in light of current design practice for SDG systems and to highlight cases where different design objectives might result in a system more conducive to sulfides generation.

By transporting septic tank effluent rather than raw waste SDG sewers can hydraulically have smaller diameter pipes placed at less steep slopes. The net result is that the wastewater travels more slowly. Also in some design approaches portions of the line may be continually full. In addition, the lack of abrasive solids limits the scouring action on the slime layer of the pipe wall. These are conditions which clearly favor sulfides generation. On the other hand, a significant portion of the sulfate may have been converted to sulfide in the septic tank and may be vented to the atmosphere in the sewer, since conditions in the latter would favor a lower (S_e). This could dramatically reduce the Sulfide Potential within the SDG system.

It is impossible to quantitatively predict the relative importance of these factors. However it is important that a designer consider the sulfate levels in the water supply to assess the overall risk of sulfide generation within an SDG sewer and then design the system accordingly.

It is evident that alternative systems of the types that have been considered will generally deliver wastewater carrying relatively high sulfide concentrations. This does not invalidate the choice of such a system, but some design modifications at the plant and of the collection system may be necessary.

Unless a sewage stream will be sulfide free, some ways to cope with the H_2S related odor and corrosion problems at the plant are necessary. Both Quaker Lake and Glide systems show that the odor problem can be solved by submerged discharging to an amply sized pond. With some other systems it may be necessary to consider some form of chemical or biological treatment for sulfide control before the wastewater enters the treatment processes. It is not within the scope of this report to transmit a comprehensive discussion of the complex subject of sulfide control technology.

SECTION 5

TWO-PHASE FLOW IN PRESSURE AND SMALL-DIAMETER GRAVITY SEWERS[1]

INTRODUCTION

In pressure sewer systems, a small diameter sewer main similar to a water main installation is buried shallowly and follows the ground contours. Each home or group of homes has a small pump. The pump may be a grinder pump (GP) which grinds the solids to a slurry, or an effluent pump septic tank combination (STEP).

Small diameter gravity (SDG) sewers are similar to STEP systems in that a septic tank is used at the home to capture the troublesome matter in sewage: grit, grease, bulky and stringy material. As the name implies, flow is by gravity, negating need for pumps. SDG systems take a variety of forms. Mains can be placed at a constant downgrade slope between cleanouts. This is similar to conventional sewer design practice, although the slope may be less steep due to the reduced scouring velocity required since septic tank effluent is conveyed. Another form of SDG sewer is like a pressure sewer main in that is can (within limits) follow the contours of the terrain. Sewage may flow in an upslope direction so long as the energy grade line slopes properly. This type of SDG sewer is sometimes called a variable grade sewer (VGS).

Technologies of STEP pressure sewers (PS) and SDG systems may be combined into one collection system.

Like gravity sewer technology, the hydraulic analyses of common domestic water supply systems apply somewhat to PS and SDG systems. However, there are important differences. One important difference, the matter of two-phase flow (air and water), is discussed herein.

ENTRANCE OF AIR

When air or gas bubbles are present in a hydraulic pipeline, flow problems can occur. To expel the air, air release valves are typically placed at summits within the pipeline. However, as explained later, this practice is insufficient when used with some configurations of PS or SDG systems.

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1. Written originally by William C. Bowne, Roseburg, OR, for USR&E.

Air may enter a pipeline in a variety of ways. A one mile water conveying pipeline contains enough dissolved air to completely fill over 100 feet of the pipe, assuming that water contains about 2 percent dissolved air by volume. The air (or gas) volume present in sewage would differ by some unknown amount from that in water, but the concept is the same. Because the solubility of gases depends on pressure, as the wastewater is conveyed through a pipeline over varying elevations, gas comes out of solution at higher points. Once out of solution, the gas will not readily redissolve.

If pump intake submergence is shallow in a pressure sewer, air may also enter the main by vortexing at the pump. For this reason, it is preferable to submerge the pump intake sufficiently. The exact depth depends on discharge rate.

When the pipeline is such that the static hydraulic gradient elevation of the main is lower than the elevation of an adjacent pump intake, there is another source of air entrance. When flow in the system is zero or minimal, the hydraulic gradient of the main is at or near static elevation. A siphoning condition develops, causing flow through the pump even though the pump is not running. This flow continues until the liquid level in the pump vault is lowered to the elevation of the pump intake, which breaks the siphon. Air then enters the service line connecting the main and the pump, and is forced into the main when the pump again operates. This is shown diagrammatically in Figure 1. An enormous amount of air may be introduced to a system at start-up, or following the repair of a ruptured main. System design should provide means of purging such large volumes of air and the methods should be described in maintenance training.

There is another source of air in SDG and PS systems. PS and SDG mains may flow "uphill" or "downhill", as shown in Figure 2. When some portion of the collection system is at an elevation higher than the point of discharge to atmosphere, flow is "downhill" within some reaches of the piping. During static conditions with downhill flow, portions of the system will drain and fill with air. The air is drawn into the main as previously described, through the main terminus, or through leaky pipe joints at zones of negative pressure.

While it is important to totally prevent the accumulation of air (or gases) within the piping system, the volume can be minimized by design, and negative effects can be attenuated.

EFFECTS OF AIR IN PIPELINES

A classic air bound pipeline is shown in Figure 3. Suppose a pump is used to move water from one reservoir to another. Water in the first reservoir is at elevation zero, and the pump has a shutoff height of 105 feet. Assume also that water in the receiving reservoir is at elevation 80, no portion of the interconnecting piping is at an elevation higher than 80 feet, the pump is running, and there is zero discharge.

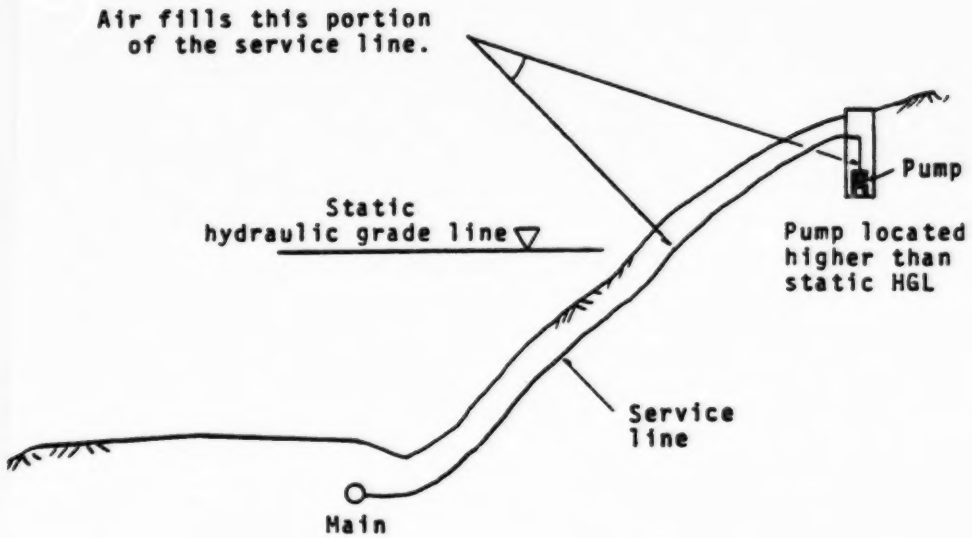
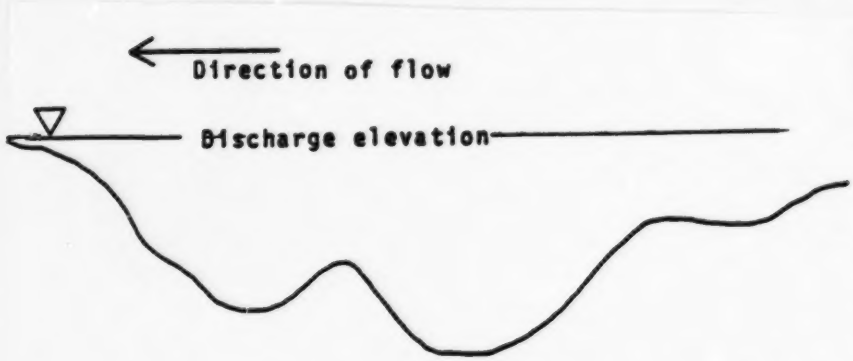
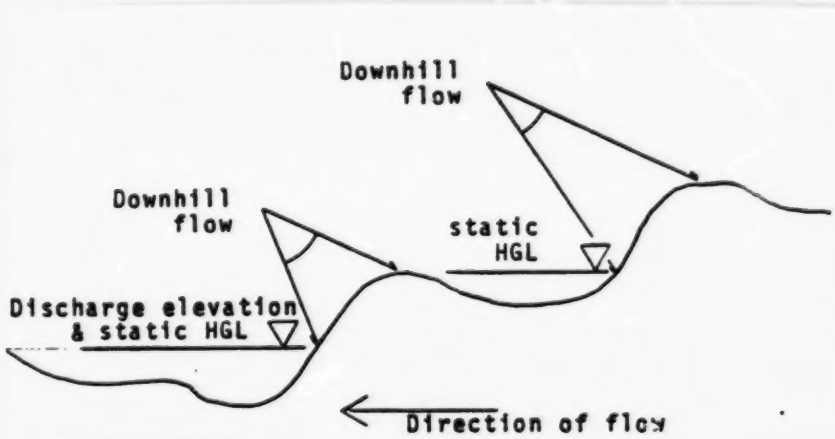


Figure 1. Cross-section at pump installation where pump is above hydraulic grade line of main.



a) "Uphill" flow. All parts of collection system are lower than the point of discharge to atmosphere.



b) "Downhill" flow in portions shown.

Figure 2. Profiles of uphill and downhill flow in sewers.

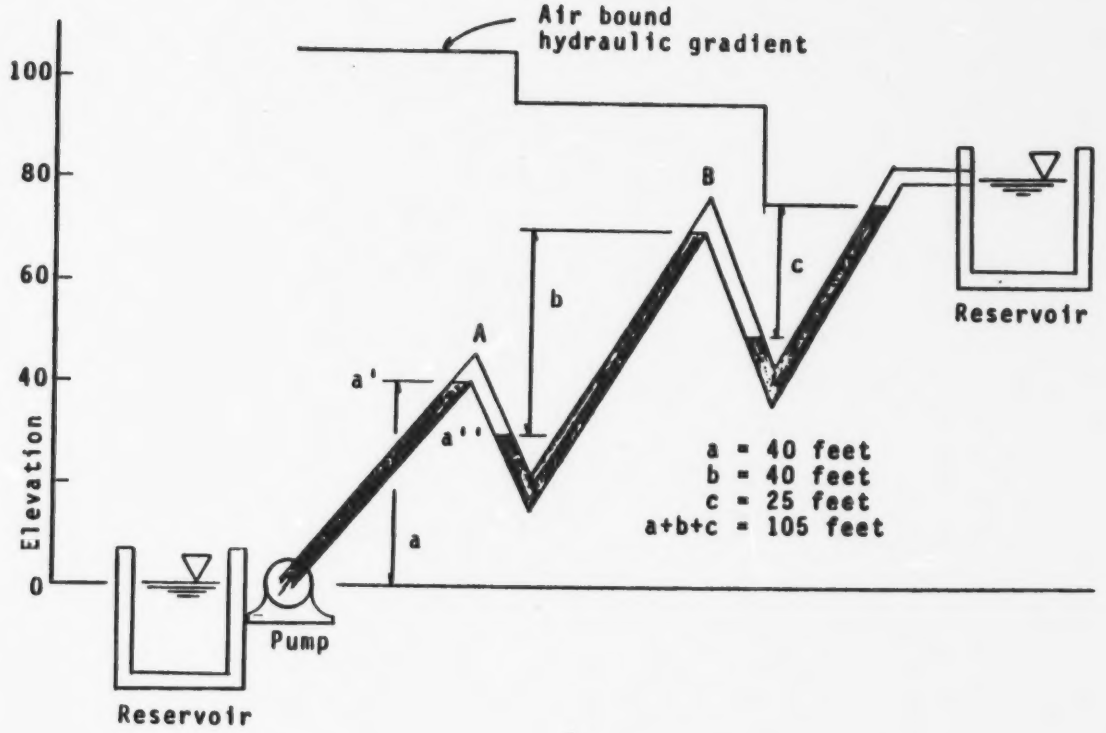


Figure 3. Air-bound hydraulic gradient.

In this example, air is trapped at two summits, A and B. The air is displaced some in the direction of intended flow, as shown. At summit A, the pressure at a' and a'' is the same (65 feet of head) even though the liquid level of a'' is ten feet lower in elevation than at a' due to displacement of the water. A similar condition exists at summit B. Summing, a, b, and c results in 105 feet, the shutoff head of the pump. No flow is occurring, and the static hydraulic grade line is not at one elevation, but is instead stepped as shown. One method of determining the presence and location of pipeline air is to create a pressure on a static main and then determine the hydraulic gradient at particular stations. This example of air binding shows how many pumps routinely operate at higher heads than necessary and points out a useful tool in hydraulic analysis.

There are other concerns in addition to air bound situations. When air is present in a hydraulic pipeline:

- o Flows are erratic, unpredictable, and have high head losses.
- o Check valves are hammered and sometimes destroyed.
- o Pumps fluctuate over a wide range of their H-Q curves.
- o Acid may form on the pipe wall, creating a corrosive situation if the air pocket is stationary.
- o Gravity flow or intermittent gravity flow may allow grease and other solids to air plate on the pipe walls.

Because the grease and other solids plating problem, GP systems are not recommended for use in downhill flow situations.

When flow is downhill and siphoning through the pump occurs, the pump itself will often become airbound. When the liquid level in the pump vault again rises to "pump on" level, air is trapped in the volute. The pump will run, but will not pump water. This condition may continue until the motor burns out, necessitating a maintenance call.

Submergence is required for two reasons. Pump and motor submergence cools equipment, prolonging life. Nonsubmergence of the motor violates explosion proof requirements (NEC 501-8(a)) (10).

PREVIOUS RESEARCH

Problems of pipeline air have been the subject of intensive design efforts on some larger systems. Typically, these pipelines convey water from mountain regions to distant population centers, and are on overall descending grades. Although the large size of the systems make them dissimilar to PS or SDG projects, the problems are similar, though of smaller magnitude.

Upstream control on a closed conduit is illustrated in Figure 4. Pipe stand and pipe check and vent structures is used to maintain a full pipeline upstream from the control structure. This practice reduces or eliminates surge and air entrainment problems.

In his doctoral thesis, Kent investigated the entrainment of air in downhill flow (11). A four-inch diameter clear plastic pipe was used, which allowed observation and scientific evaluation.

When air pockets form in the pipeline, as shown in Figure 5 and the initial depth of flow is not below critical, a hydraulic jump cannot be formed. Nevertheless, the action below an air pocket closely resembles a jump. The jump is violent, with associated high head losses, and bubbles are ripped from the pocket. The bubbles move downstream, with smaller bubbles traveling faster than large ones. The bubbles tend to join, and form another pocket from which bubbles are again torn, and the process continues. The pocket may be stationary, move upslope, or move downstream. Depending on the diameter and slope of the pipe, Kent developed equations below to describe the velocity required for the pocket to remain stationary (the point where drag and bouyant forces are equal). The velocity required to attain this condition is a direct function of pipe diameter and pipe slope from the horizontal plane

$$V_{min} = C_o^{1/2} \ ((gD \sin \theta)^{1/2}$$

where V_{min} = the minimum velocity to keep an air pocket stationary, ft/sec.

$C_o^{1/2}$ = constant when the air pocket length is 1.5 pipe diameters, dimensionless.

g = acceleration due to gravity, 32.2 ft/sec^2

D = pipe diameter, ft.

θ = angle of pipe from horizontal

V_{min} in PS or SDG applications will typically range from 1/2 to 2 feet per second, the higher values related to larger diameter pipes and steeper slopes. In most PS and SDG systems, movement to the terminus of the pipeline will not purge the air because flows of sufficient velocity are not of long enough duration. The air will move only a limited distance downstream and return upslope when the peak flow condition has passed.

The equation for head loss due to the presence of air in the pipe developed by Kent indicates that head loss is directly proportional to the percentage of air by volume and slope. Therefore, head loss increases with quantity of air and as pipe slope increases. A few calculations show that head losses due to air can be large and of substantial importance to

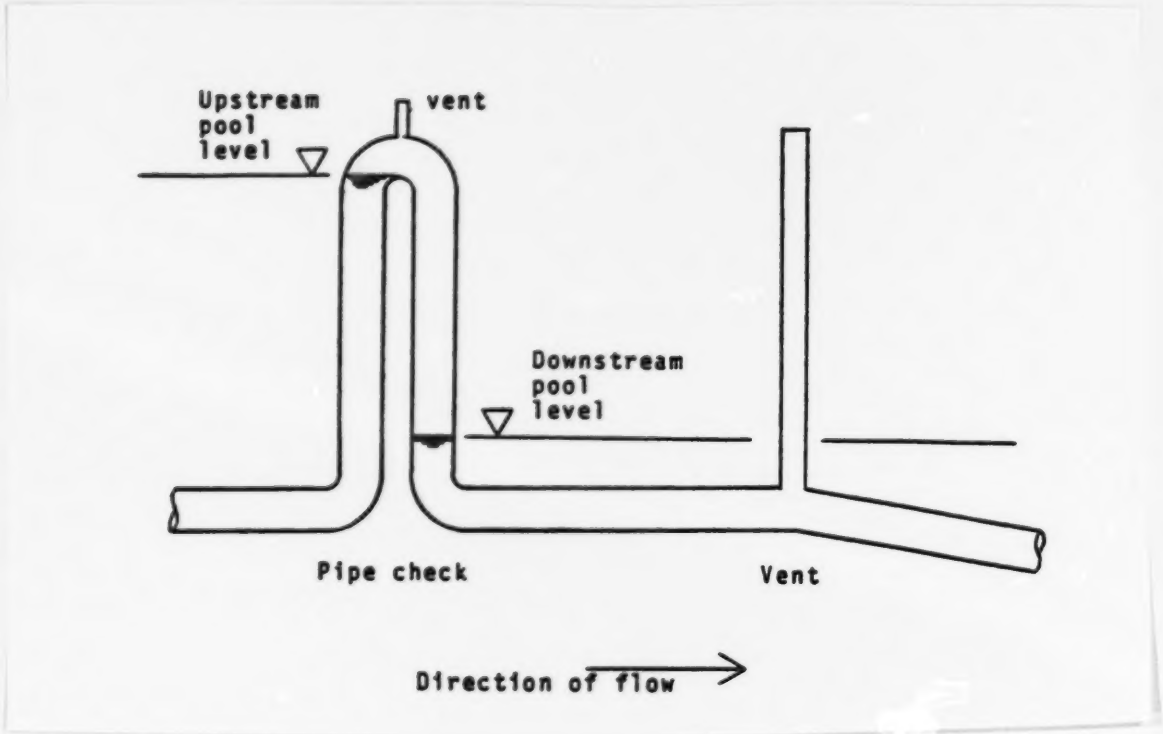


Figure 4. Pipe check and vent.

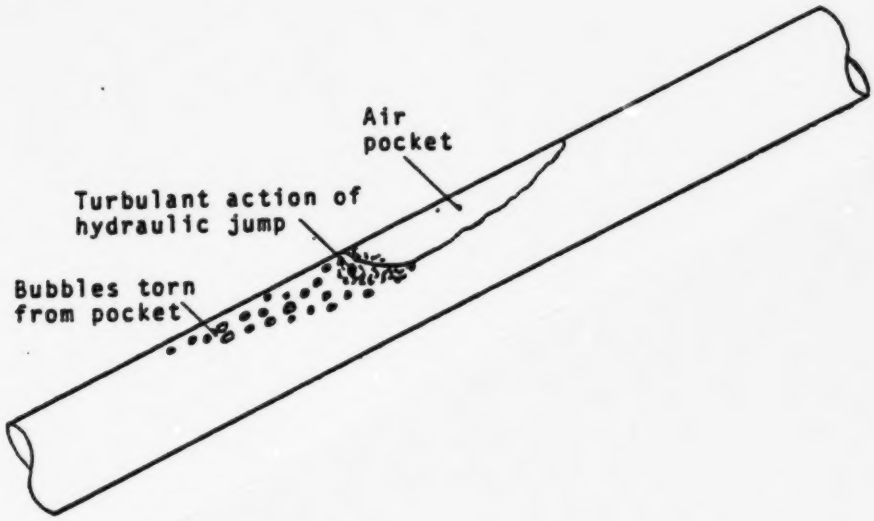


Figure 5. Air pocket and bubbles encountered in downhill flow.

PS or SDG design. For example, a one percent volume of air, with a 10 percent (5.7 degree) pipe slope would correspond to a head loss of 0.001 ft./ft. A head loss of 0.01 ft./ft. would occur at 10 percent air. Recognizing that these head losses would be additive to losses determined by hydraulic equations, we see that they can easily become excessive.

LABORATORY AND PILOT EXPERIMENTS

To better understand the phenomena of two-phase flow, several experiments were performed.

A 1-inch diameter transparent plastic tube, about 30 feet long, was fastened to wall-mounted pegboard with hooks. The hooks could be easily adjusted to alter the pipeline profile. Tees fitted with vertically rising 1/2-inch clear tubing were placed periodically in the tubing. The 1/2-inch tubing served as piezometers or as air release points, depending on location. A pressurized water supply was connected and a venturi aspirator used to inject dye to make the flowing water (and air pockets and bubbles) more visible, and to introduce air to the pipeline.

The experiment was operated for several hours daily over a period of about two weeks. When flow was uphill, the system ran quietly and predictably. When air was introduced to the system, it was effectively expelled by the air release vents. At high flow velocities, some air bubbles would pass the vents, but the volume was small, and no particular difficulties were noted.

It was a different matter when flow was downhill. The larger volume of air created by the draining of portions of the tubing changed the hydraulics completely. Pockets and bubbles similar to those described by Kent formed. The hydraulic jumps were violent, while flow was highly erratic, unpredictable, and accompanied by high head losses. Water would suddenly spurt out of the piezometers and air release vent tubing. The hydraulic grade line was much steeper than when flow was uphill. Under uphill flow conditions, water would never spurt from the vertical tubing, nor would the hydraulic gradient rise to near the top of the tubing, even at much higher flow rates.

This was a worthwhile experiment, easy to construct and operate. It is a recommended experience for engineers contemplating two-phase flow design.

Because of the small diameter of the experimental tubing, the experiment was not entirely representative of PS or SDG sewer application, so another test was conducted. In this experiment we used 100 feet of 6-inch diameter transparent plastic piping, sloped at 2.5 percent, and flowing at 185 gpm (about 2 feet per second velocity). Discharge was to a receiving vessel with a liquid level at the elevation where the static hydraulic grade line intersected the sloping pipes (See Figure 6). During static

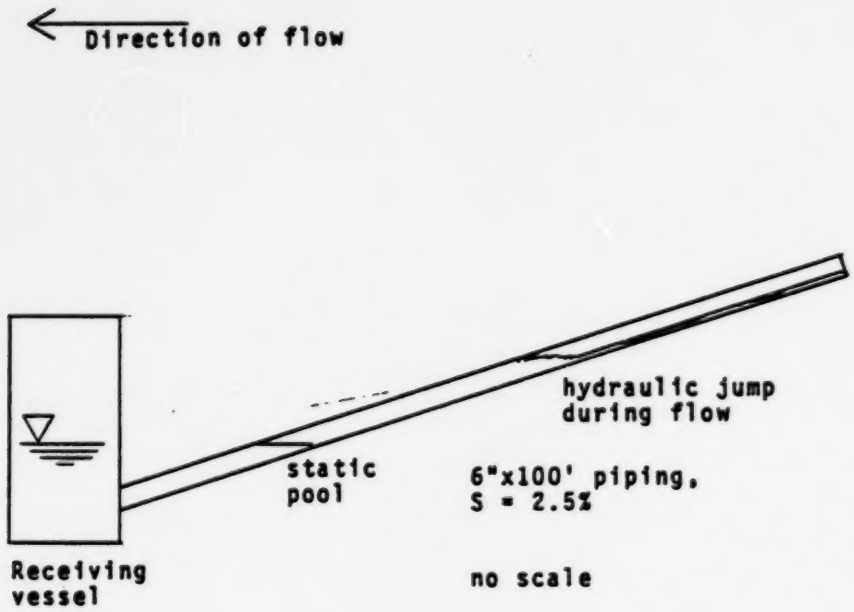


Figure 6. Test apparatus using transparent piping.

conditions, the length of the ellipse formed by the water surface in the pipe at the static HGL was about 20 feet long. When flow began, the jump would develop about 3 to 4 feet upstream from the toe, or most downstream point of the ellipse.

Except for a few parameters, results were fairly similar to those obtained in the previous test and as described by Kent. Bubbles traveling downstream would often pass pockets rather than joining them. Very small bubbles (approximately 2 millimeters diameter) seemed to move downstream at a slower effective rate than larger bubbles, because the small bubbles took a more erratic path, darting about as they traveled.

Air bubbles and pockets were effectively removed by a 1/4-inch diameter hole drilled in the crown of the pipe. When the hole was placed 14 diameters below the jump, the vent was an estimated 95 percent effective in removing air. Effectiveness did not increase at a greater distance downstream.

FIELD STUDIES

This work was conducted to advance the knowledge of PS and SDG hydraulics generally and to assess the design of the Glide, Oregon pressure sewer system. Glide is the world's largest STEP system, and combines a few SDG connections (4). Topographic conditions made downhill pumping manadatory in some instances, and it was necessary to provide the most reliable hydraulics possible. The system is sized to serve 7,300 people, and initially serve 2,000. Predicted peak flows were 350 gpm initially and 1,050 when the system was fully operational.

Much of the collection system had been installed at the time the laboratory experiments were conducted, yet design refinements were continuing. It was decided to investigate the hydraulics of two-phase flow in a two-mile portion of the main.

The test section involved 2,350 lineal feet of 12-inch diameter main, and 8,270 feet of 10-inch main. The maximum elevation difference of the pipeline profile was 58 feet. Two automatic air release valves (AARV) were a part of the original installation, but by the time testing was completed, one more had been added, and one relocated, for a total of 3. The ARRV's were sewage-type air release valves, not vacuum or combination, with 5/16 inch orifice.

Flows of from 200 gpm to 700 gpm were produced by pumping river water with a diesel fuel powered pump. Flows were measured using an impact tube meter (Metraflex™ model 23) and checked with reasonable agreement using the XY coordinate method calculation of water drop and the pump curve. Pressure recording stations were located at four points along the main. Laboratory test gauges were used, the accuracy checked daily with a mercury manometer.

The main was slowly filled with water over a period of two days, and air purged. A plot of the profile of the main is shown in Figure 7. Drawn on that plot are the static hydraulic grade lines, as well as the theoretical dynamic hydraulic gradeline based on the Hazen Williams equation using an assumed C value of 140 and a flow rate of 600 gpm. The assumption was temporarily made that flow would break to gravity flow in the steeply sloped pipe reach between stations 70 and 80. In developing the dynamic gradient, no allowance was made for headlosses due to air. The dynamic HGL was drawn to allow a quick visual comparison between actual conditions and theoretical conditions without allowances for air.

The system was then slowly started up, and the air release valves constantly checked. The system was operated for a full day at flows ranging from 200 to 600 gpm. As expected, headlosses were high. The elevation of the hydraulic gradeline was determined from pressure readings at the observation station. A typical HGL for the day, at a flow rate of 600 gpm is shown in Figure 8. The HGL has been drawn as a straight line between pressure stations for ease in interpretation, but in actuality the headlosses would vary within each reach and the true HGL would take on other shapes. At 600 GPM, notice how much higher the headlosses actually are than hydraulic calculations would predict without accounting for air. Realize that this two-mile test section was typical of the 25 miles of main comprising the system. Such a system that ignored air considerations in the design would not operate suitably.

The shutoff valve at station 0 was then slowly closed while the pump was still running, and pressure readings taken at the stations at one minute intervals. The close time intervals were necessary since air made the static pressures constantly change. A typical plot is shown in Figure 9. Note that the pressurized static HGL does not attain a single elevation. This reveals trapped air, as seen in the example in Figure 3. Clearly, air was trapped in the reach between approximate stations 70 and 80. No doubt much air was also contained between pressure station 1 and 2, but the fairly flat shape of the pipeline profile between these points prevented the pressure readings from revealing its presence.

Next, the piping system was modified. The air release valve near station 81 which had been placed on the exact summit of the pipeline was relocated to a point several hundred feet downstream. An automatic air release valve was also added at station 69.

These changes had been anticipated from experience in operating model studies. The reason for the change at station 81 is that under static conditions, water would stand to the right of the invert summit as shown in Figure 10. As flow began, the profile of water surface would soon seal off the entrance to the air release at the summit. Yet, as flows increased and the hydraulic grade line rose in the reach between stations 70 and 80, over 4,000 gallons of water (septic tank effluent) filled the reach. The displaced air had to be expelled.

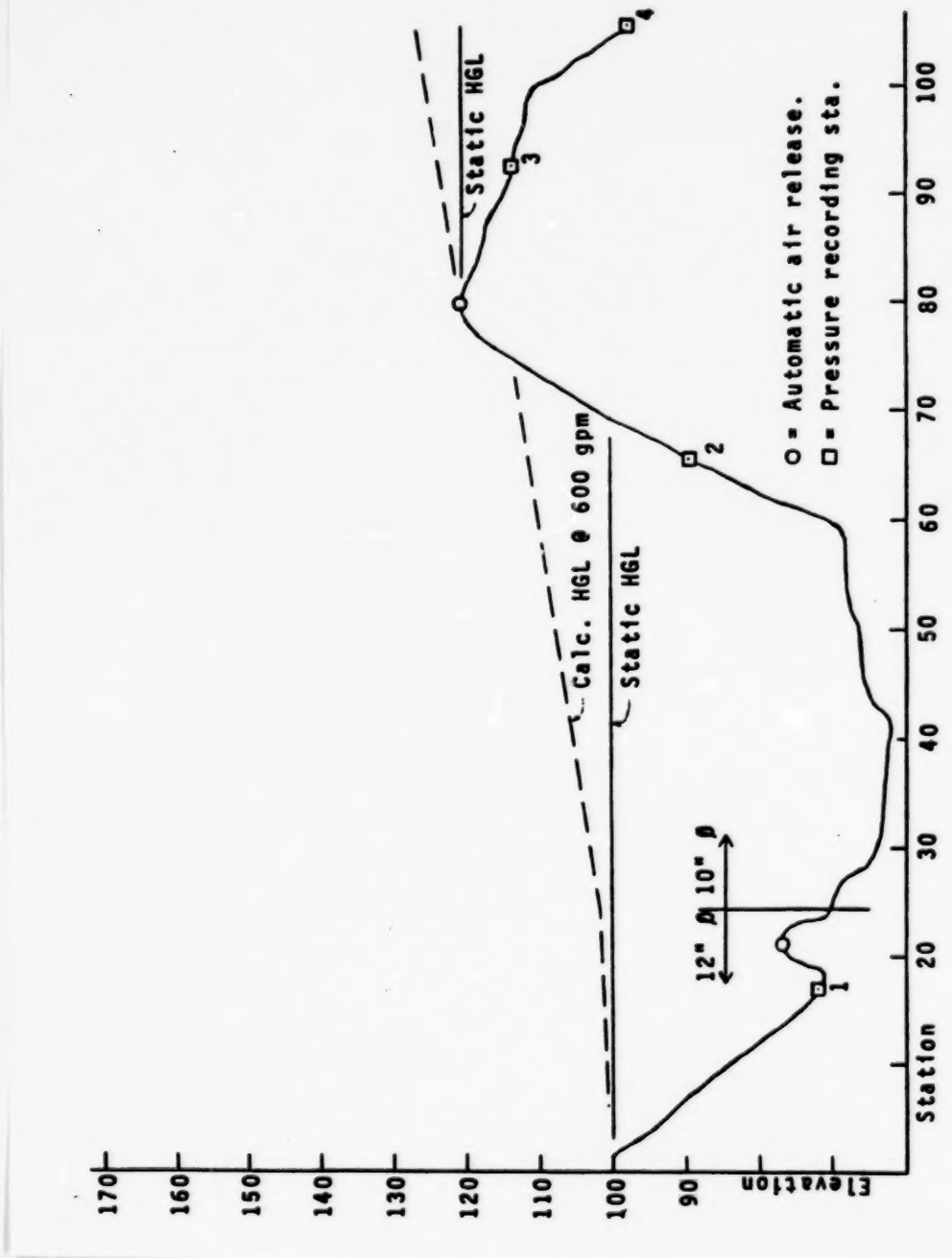


Figure 7. Profile of two-mile test section of Glide pressure sewer system, ignoring air considerations.

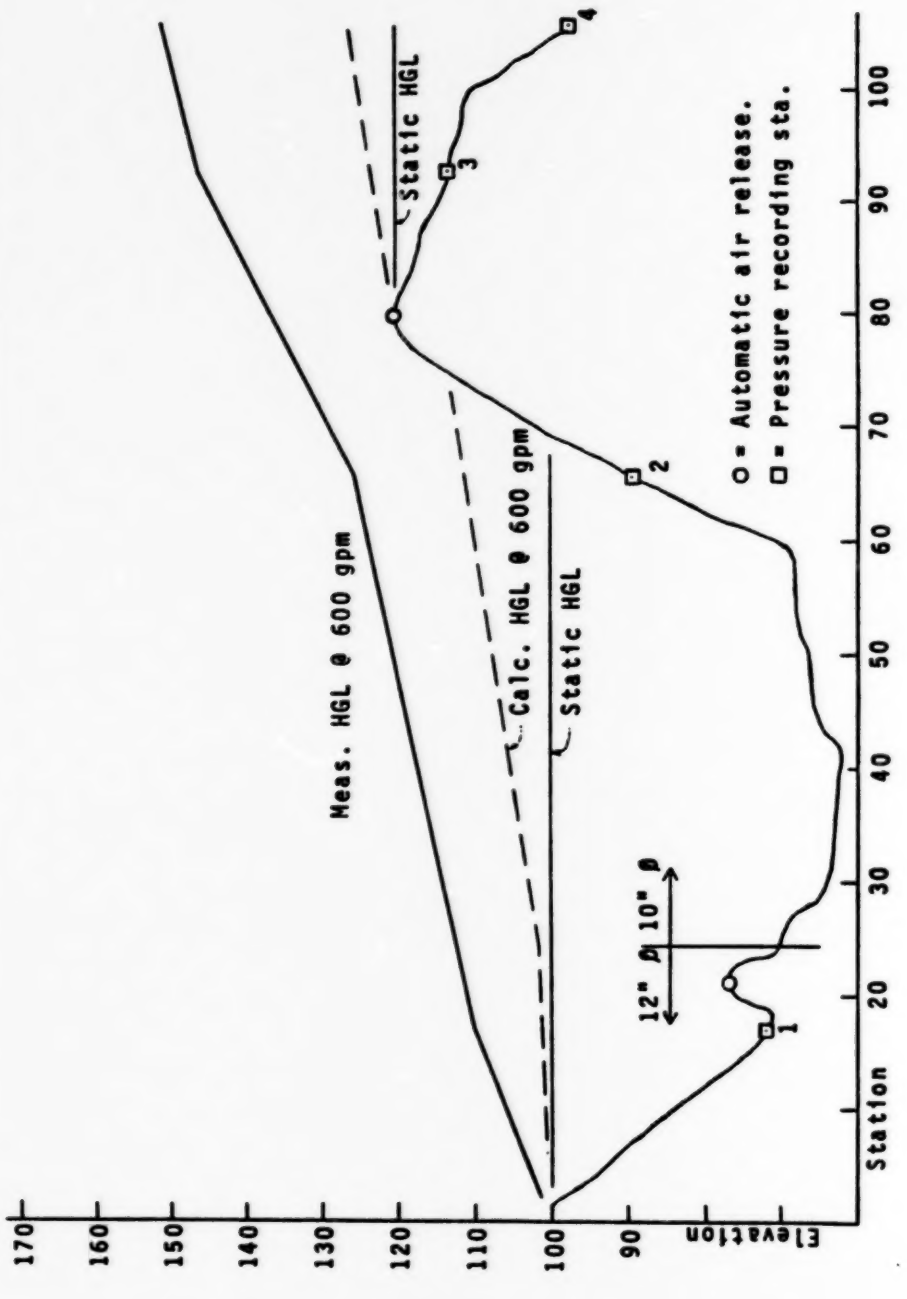
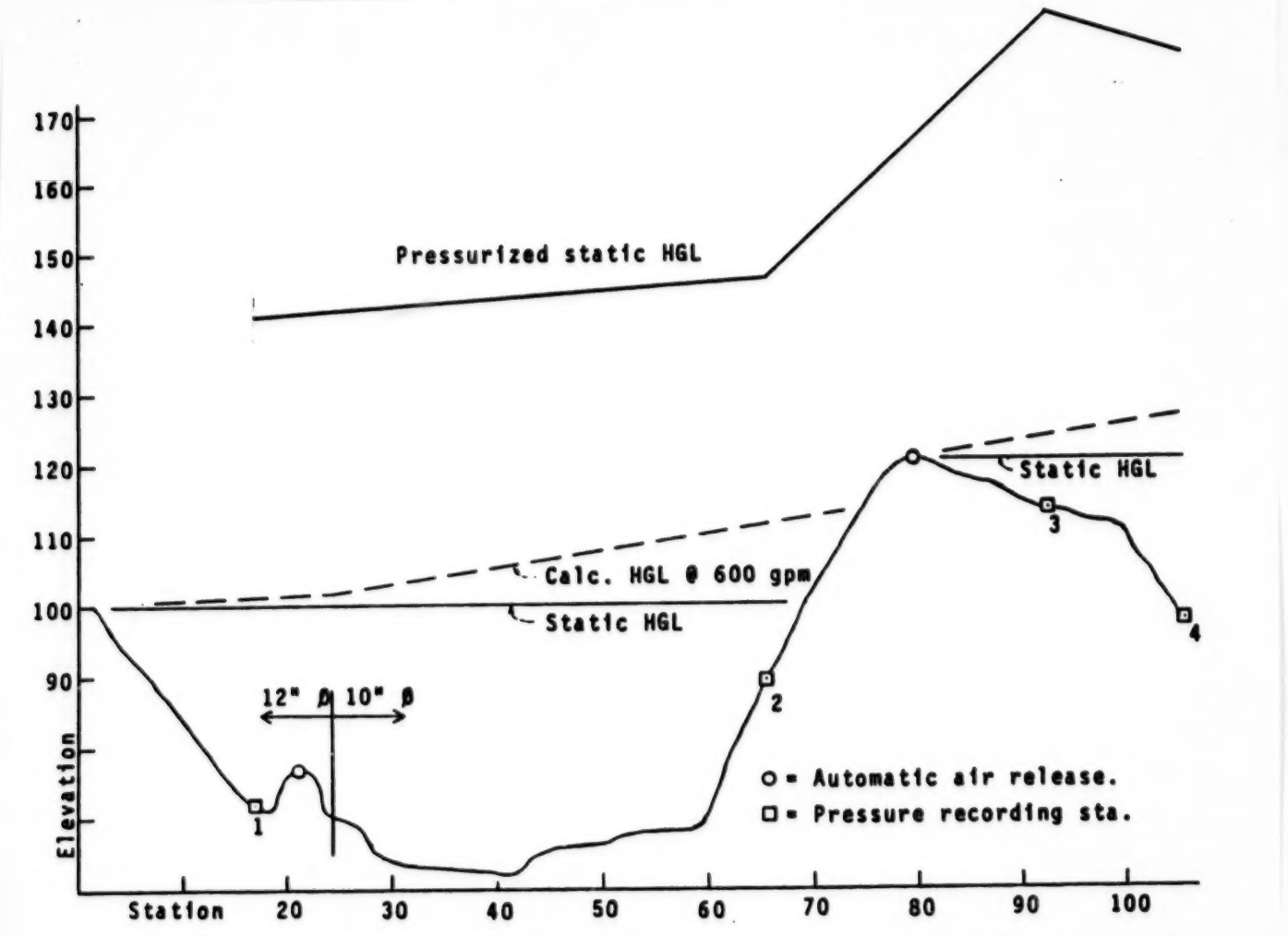


Figure 8. Measured hydraulic grade line at 600 gpm.


Pressurized static HGL
Static HGL
Calc. HGL @ 600 gpm
Static HGL
12" Ø 10" Ø
1
2
3
4
○ = Automatic air release.
□ = Pressure recording sta.
Elevation
170
160
150
140
130
120
110
100
90
Station
20
30
40
50
60
70
80
90
100

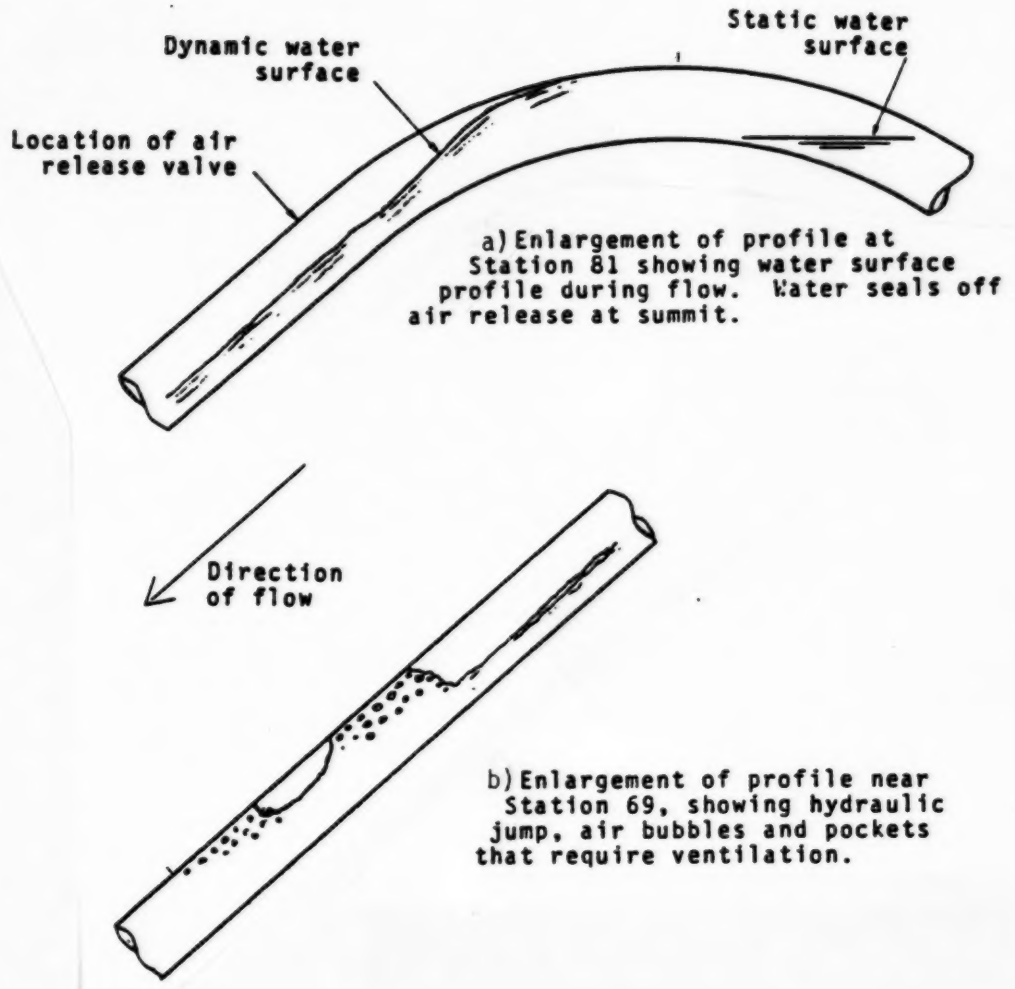

Static water surface
Dynamic water surface
Location of air release valve
a) Enlargement of profile at Station 81 showing water surface profile during flow. Water seals off air release at summit.
Direction of flow
b) Enlargement of profile near Station 69, showing hydraulic jump, air bubbles and pockets that require ventilation.

Figure 10. Pipeline profiles at stations 69 and 81.

The situation at station 69 is shown in Figure 10. Under flow, a hydraulic jump is formed where the dynamic hydraulic grade line intersects the main. The jump is violent and pumps air bubbles into downstream reaches. To capture and ventilate that air, the connection of the air release assembly to the main must be made below the jump.

Once installation of the air release valves had been made in the revised locations, flow at various rates was again introduced to the system. Time was required to work the air out of the pipeline. During this period, gauge readings were taken at the pressure stations. Typical results are shown in Figure 11. Note that the HGL is much lower than in Figure 8, and as time progressed with flow occurring, the HGL continued to drop and become of flatter slope, approaching the theoretical.

DESIGN OF THE GLIDE, OREGON SYSTEM

A portion of the Glide piping system was designed and now operates as previously described, and as shown in Figure 11. Another portion of the collection system had a different profile, characterized by downhill runs and multiple locations of hydraulic jumps that were less steep and distinct. Treatment of the situation by installation of air release valves as used at stations 69 and 81 was not practical.

The type of profile referred to is shown in Figure 12. To obviate the problems of downhill flow, standpipes were used to keep upstream reaches full. The standpipe design is shown in Figure 13, and its application shown in Figure 14. The PVC standpipe was located in an area of relatively steep topography so that the it could be buried at the usual depth and run a short distance to attain the desired elevation at the crest to keep the upstream reaches submerged. A hydraulic jump occurs in the downleg of the standpipe. This downleg is steeply sloped, and of larger diameter to reduce flow velocity and prevent the conveyance of air bubbles into the downstream reaches. Air release valves were used to the exact summits of the pipeline upstream from the standpipes.

An investment of several months engineering time was made to investigate the use of special backpressure sustaining valves in lieu of standpipes. Despite special attention given to the design and selection of the valves by valve company engineers, none worked properly. A variety of problems developed. In some cases, the valve opening and closing pressures differed too much for them to be used. Some valves would not close completely enough, especially if small amounts of solids were caught on the valve seat. Other valves would constantly hunt for position, while still others were so complicated and expensive that they were impractical.

Valve failure modes were undesirable. Neither failing open nor failing closed were suitable options. The standpipes were passive and accomplished the goal of maintaining full upstream reaches simply, reliably, and economically.

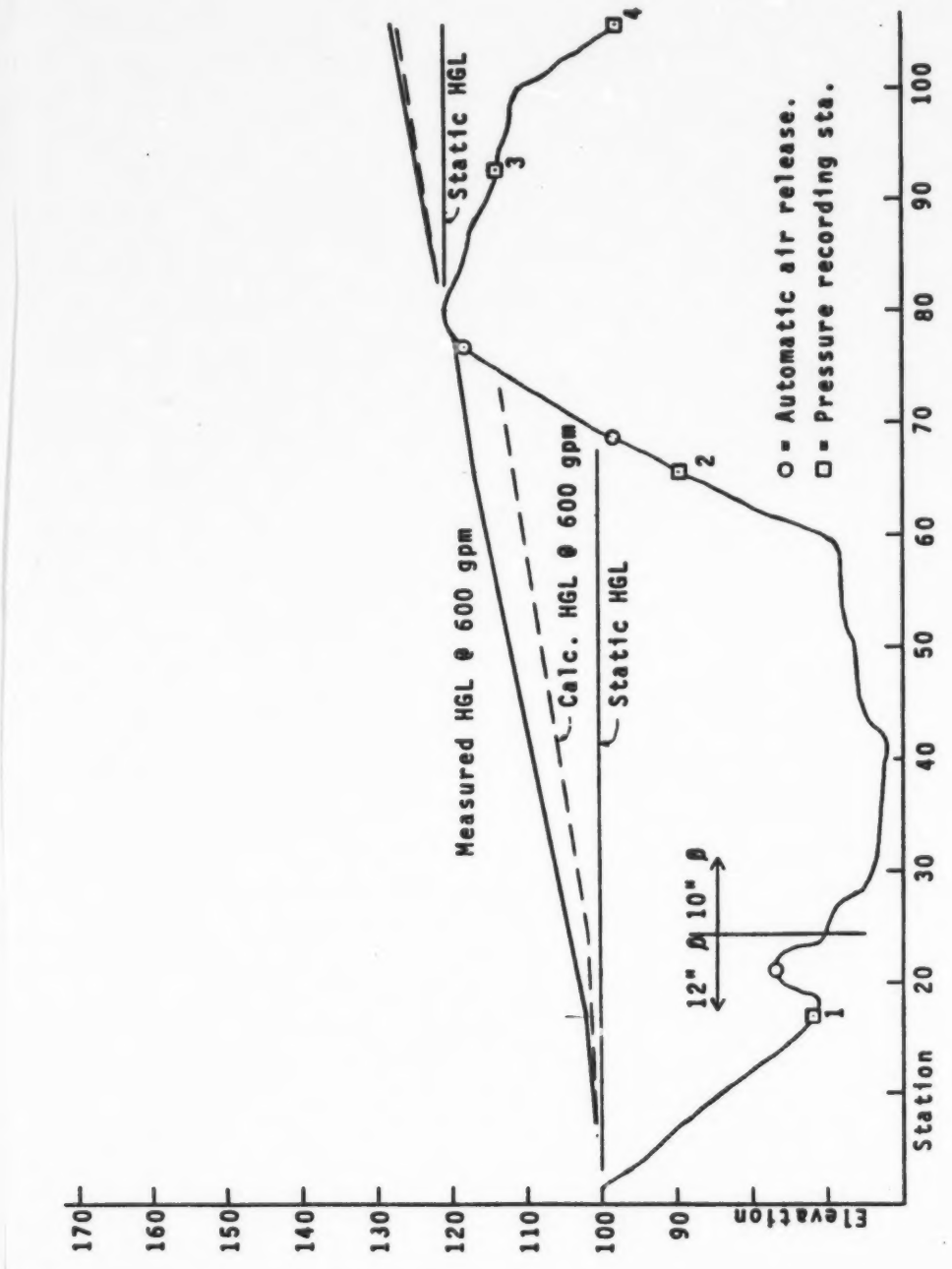


Figure 11. Measured HGL at 600 gpm after alterations.

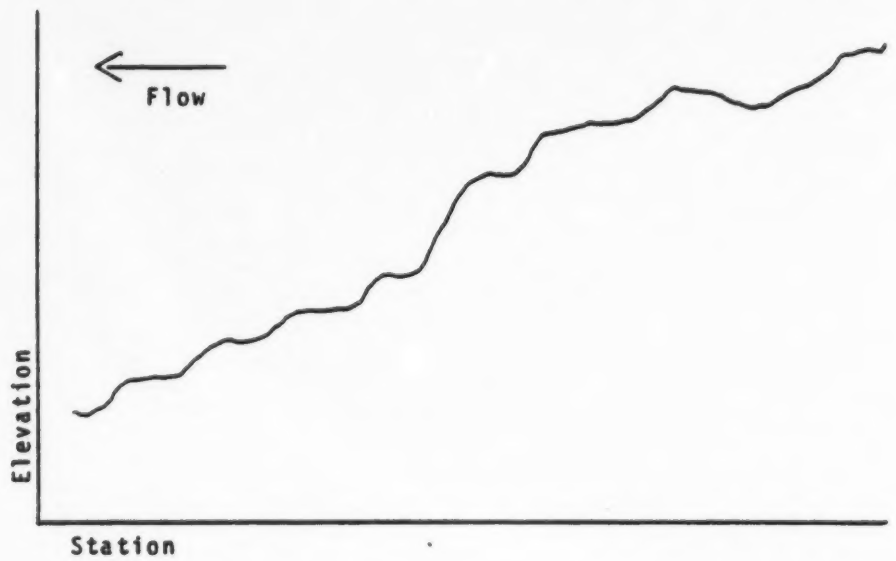
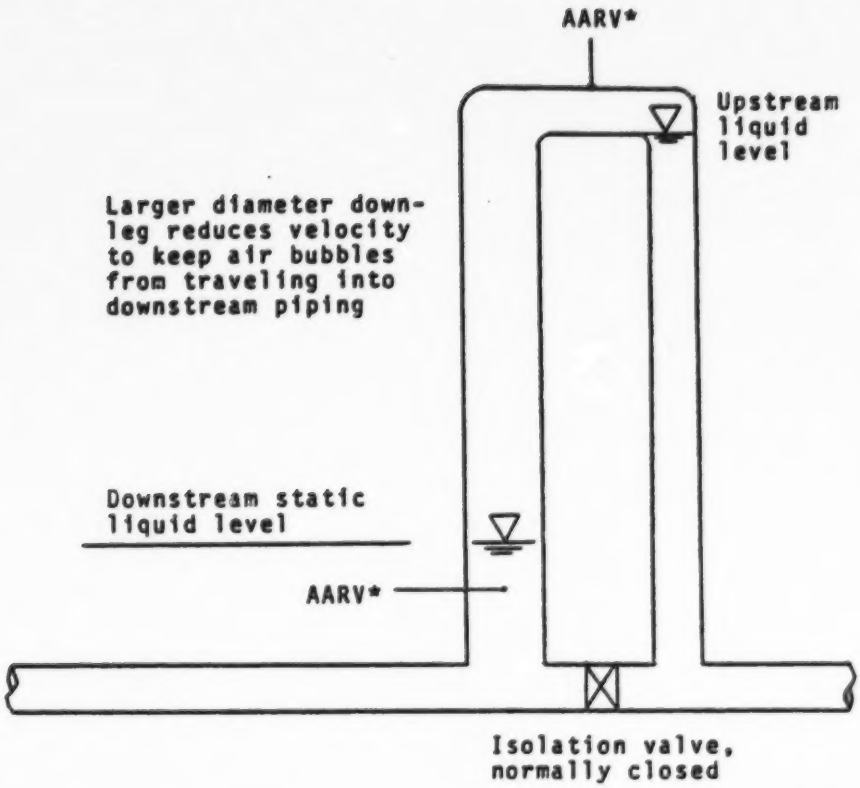


Figure 12. Pipeline profile with numerous downhill flow sections.


AARV*
Upstream liquid level
Larger diameter down-leg reduces velocity to keep air bubbles from traveling into downstream piping
Downstream static liquid level
AARV*
Isolation valve, normally closed

AARV* = Automatic air release valve vented to soil bed for odor absorption

Figure 13. Standpipe.

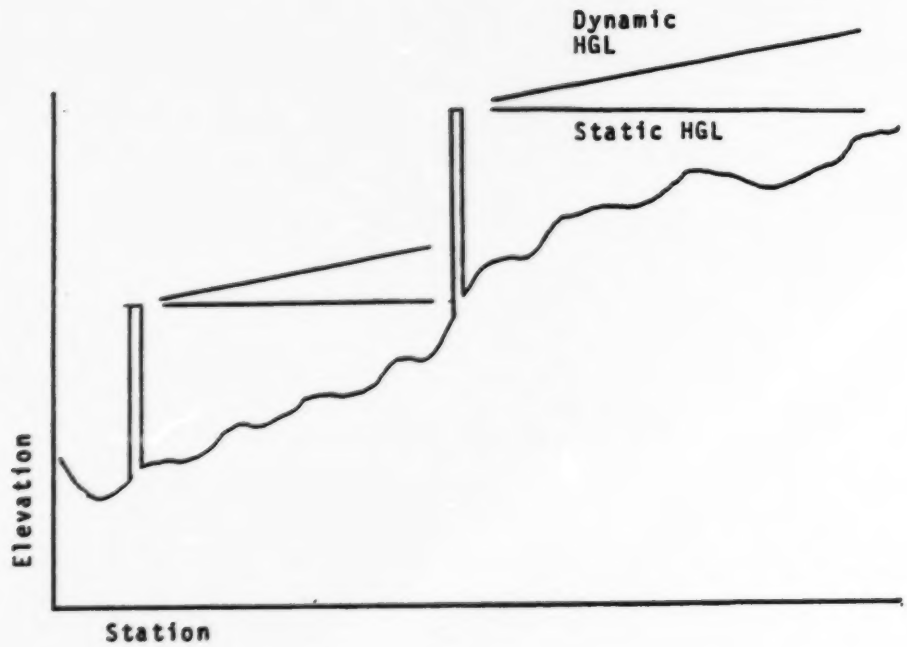


Figure 14. Pipeline profile using standpipes, showing static and dynamic HGL's.

As air release valves are normally used on a force main, only small volumes of air are expelled. Large volumes are expelled at installations such as at station 69, 81 and the standpipes. Since odor from hydrogen sulfide gas was expected, the air release valves were vented to soil beds for odor absorption (4).

When a septic tank was located at an elevation higher than the dynamic HGL, a gravity installation was used. The tank was fitted with a 1-1/4 inch diameter outlet tee, located at an elevation lower than would normally be found in septic tank practice. The purpose of the lowered level was to provide reserve volume within the tank. In the event valves on the pressure sewer main were closed to allow for repairs on the main, flow from the home could continue for at least one day before the tank overflowed.

The gravity service line was 1-1/4 inch PVC, placed without regard to grade, the same as used on the STEP installations. A check valve was used on the service line at the main in those cases where a STEP pump could pump a gravity tank if an isolation valve on the main were closed.

OPERATING EXPERIENCE WITH THE GLIDE SYSTEM

Over three years have passed since the Glide, Oregon system became fully operational. During this time, performance has been carefully monitored. Air release stations are inspected frequently. Pressure readings are taken at designated stations throughout the system, and compared with previous charts.

The automatic air release valves have proven reliable. Rarely does one malfunction, and never to cause any difficulty, contrary to the general reputation of sewage air release valves. This performance is attributed to the low grease and solids content characteristic of septic tank effluent.

No odor has ever been detected at air release stations vented to soil beds. It is not known if odor problems would exist if the valves were vented in the usual way, but it is logical to expect they would.

Peak flows are on the order of 350 gpm. Since the system is sized for 1,050 gpm, it operates far below its ultimate capacity. However, on two occasions the service area was flooded, the standing water on the ground surface entered the pressure sewer pump vaults, and peak flows to an estimated 1,200 gpm occurred at those times. Some of the pressure sewer pumps were dominated by others to shut off head during these periods as would be expected since flow exceeded design. The fact that design flows could be exceeded testifies that the system was performing properly.

There has been no evidence of air plating problems, or any other problem with the collection system piping. Some treatment occurs in the pipeline, but it is unknown how much can be attributed to air entrainment at the hydraulic jumps.

There are a few places where pumps are above the static hydraulic gradient. These pumps occasionally became air bound. In one case, a pump was located at an elevation substantially higher than the static HGL, and higher than the dynamic gradient. About weekly, the service line at this installation became air bound as shown in Figure 3, and the pump would be driven to shutoff head. No problems have occurred since the installation was switched to a gravity connection. The trickle gravity flow from the septic tank to the service line allows air to escape from the service line back through the tank outlet tee rather than becoming trapped by the sudden and larger flow produced by a pump.

The gravity installations have had no problems. Concerns that zooglea or other matter might plug the service lines have proven unfounded so far. Evidence indicates that the effluent level in the interceptor tanks has risen perhaps a foot at times. Overflowing of the tanks has not occurred, so apparently this small rise in liquid level has developed enough head to overcome whatever restriction developed in the service line.

When flow is "downhill" in a closed conduit, such as a pressure sewer or a small-diameter gravity sewer, a hydraulic jump is formed at the intersection of the pipe and the dynamic hydraulic gradient. Bubbles are ripped from air pockets and conveyed further into the piping system. The presence of air (or gas) causes flow to occur in ways that would not be predicted by standard hydraulic equations such as Manning or Hazen-Williams. Head losses are high, and flows and pressures are erratic.

By proper design, systems can be engineered to flow downhill and avoid air problems. The techniques are not in common knowledge. An air release station is placed a short distance downstream from the jump, and at the upper reach to vent the air displaced. Standpipes may be used to locate the jump in a steeply sloped pipe section that may be of larger diameter to produce a low flow velocity.

Pressure and gravity sewers conveying septic tank effluent have been effectively combined on the Glide, Oregon project using hydraulic techniques herein described.

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